Community Noise Levels in Rochester

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16 August 2001

Mr. Stephen Thornhill
Burns & McDonnell
9400 Ward Parkway
Kansas City, MO 64114

Subject: Community Noise Levels in Rochester
DM&E Environmental Impact Statement

Dear Mr. Thornhill:

Mr. M. Amato of WIA took two spot noise samples in Rochester during his trip to Minnesota this year to measure ground vibration from passing DM&E trains. Mr. Amato was accompanied by Mr. Bowers of Burns & McDonnell. The data were collected with a Bruel and Kjaer Type 1 Precision Sound Level Meter and DAT digital recorder. The data were analyzed in our laboratory with a GenRad 1926 Real Time Analyzer interfaced to one of our laboratory computers to determine the statistical variation of the noise level over the sample durations. (The analyzer provides 1/3 octave band levels in addition to the A-weighted levels provided here, and can made available to you if you wish.) The measurements were conducted in the parking lot of Charlton North building of the Mayo Clinic south of 2nd Street NW. The measurements were at 10 am and 11 pm on June 5, 2001. The sample lengths were 10 minutes each. The results are summarized in the following table.

The Leq is the Energy Equivalent Level, used for estimating the Day Night Level, Ldn. and other related descriptors. For continuous sound levels, the Leq is the most representative. The Lmax is the maximum observed sound level. The levels L10, L50, L70, and L90 are the levels exceeded 10, 50, 70, and 90 percent of the time. The L90 is representative of the background sound level, while L50 is the median level. The L50 is a robust indicator of typical sound levels.

Table 1 Community Noise Levels at Mayo Clinic Charlton North,, June 5, 2001

Level	Day	Evening
Leq	59	54
Lmax	74	66
L10	61	54
L50	57	52
L70	56	52
L90	55	51

The levels may be expected to vary considerably from hour to hour, and day to day. The median L50 and background L90 levels would likely vary less, however, than the Leq or L10. This is due to the number and types of vehicles that might pass the measurement location along 2^{nd} St. SW. These level, however, appear to be reasonably uniform, with little disparity between the Leq, L10, and L50.

None of these data include train noise, though the samples were taken after train passage.

Please contact me if you have any questions, or desire further information.

Very truly yours:

WILSON, IHRIG & ASSOCIATES, INC.

James T. Nelson

James T. Nelson, Ph.D., P.E.
Vice President

Community Noise Levels in Mankato

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22 August 2001

Mr. Stephen Thornhill
Burns & McDonnell
9400 Ward Parkway
Kansas City, MO 64114

Subject: Community Noise Levels in Mankato
DM&E Environmental Impact Statement

Dear Mr. Thornhill:

Mr. M. Amato of WIA took two spot noise samples in Mankato during his trip to Minnesota this year to measure ground vibration from passing DM&E trains. Mr. Amato was accompanied by Mr. Bowers of Burns & McDonnell.

The data were collected with a Bruel and Kjaer Type 1 Precision Sound Level Meter and DAT digital recorder. The data were analyzed in our laboratory with a GenRad 1926 Real Time Analyzer interfaced to one of our laboratory computers to determine the statistical variation of the noise level over the sample durations. (The analyzer provides 1/3 octave band levels in addition to the A-weighted levels provided here, and can made available to you if you wish.)

The measurements were conducted west of the track alignment at about 20 feet from the track, near the intersection of Waterfront Drive and Main Street. The times of each sample were early afternoon at about 2pm on June 4, 2001, and early evening at about 7:30pm on June 5, 2001. The sample lengths were 10 minutes each. The results are summarized in the following table.

The Leq is the Energy Equivalent Level, used for estimating the Day Night Level, Ldn, and other related descriptors. For continuous sound levels, the Leq is the most representative. The Lmax is the maximum observed sound level. The levels L10, L50, L70, and L90 are the levels exceeded 10, 50, 70, and 90 percent of the time. The L90 is representative of the background sound level, while L50 is the median level. The L50 is a robust indicator of typical sound levels.

Table 1 Community Noise Levels at Mankato, June 2001

Level	Day	Evening
Leq	64	60
Lmax	76	75
L10	67	63
L50	61	57
L70	59	56
L90	58	54

The levels may be expected to vary considerably from hour to hour, and day to day. This is due to the number and types of vehicles that might pass the measurement location. The median L50 and background L90 levels would likely vary less, however, than the Leq or L10. These levels, however, appear to be reasonably uniform, with little disparity between the Leq, L10, and L50, suggesting reasonably steady traffic noise.

None of these data include train noise.

Please contact me if you have any questions, or desire further information.

Very truly yours:

WILSON, IHRIG & ASSOCIATES, INC.

James T. Nelson

James T. Nelson, Ph.D., P.E.
Vice President

Evaluation of Liquefaction Potential of Soils from Train-Induced Vibration Loading for the DM&E Railroad Project at Mankato, Minnesota Site

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Burns & McDonnell
SINCE 1898

Title: Evaluation of Liquefaction Potential of Soils from Train-Induced Vibration Loading for DM&E Railroad Project at Mankato, Minnesota Site

Prepared By: Burns & McDonnell Engineering Company

For: Expansion of DM&E Railroad Project at Mankato, Minnesota
Burns & McDonnell Project No. 24554-3.50

Date: October 2, 2001

This letter report discusses the potential for localized soil liquefaction and subsequent liquefaction-related impacts upon wayside structures and land uses, resulting from a combination of elevated ground water levels during flooding events and active train-induced vibrations. These issues are being reviewed because of the proposed expansion and upgrade of the DM&E Railroad and introduction of unit coal trains in Mankato, Minnesota. The report includes a general discussion of soil liquefaction, characteristics of train-induced vibrations, and a site-specific evaluation of the susceptibility to train-induced soil liquefaction.

In evaluating the likelihood of damaging effects to structures and nearby buildings from the proposed railroad expansion and upgrade, questions have been posed to the potential for liquefying soils that support either the floodwall or adjacent buildings. Under normal rail operation (the majority of the time), prevailing ground water in the underlying soils will reside substantially deeper below "vibration stress zones", to where liquefaction potential is not a concern. It is only during those limited occasions in which the ground water is elevated, or underseepage pressures exist due to extreme flood events, that this potential for liquefaction even remotely exists.

There are numerous adverse geotechnical-related phenomena, besides liquefaction, which may be initiated due to the occurrence of a major flood event within the project site region. It is important to distinguish the differences between each of these geotechnical phenomena, and to selectively characterize and differentiate them from the evaluation of liquefaction potential. Specific geotechnical-related phenomena that may occur adjacent to a levee/floodwall reach of a river undergoing flood events include:

- Soil piping, or boiling of soils, due to excess underseepage pressure: This may occur from an unbalanced hydrostatic head associated with elevated underseepage pressures in the vicinity of the levee/floodwall area. Initially, during flooding events, the water pressure on the landward side of a levee/floodwall system will attempt to parallel the river's rise in a slightly delayed gradient. If the pressure at any time or location along the levee/floodwall reach were to exceed the confining weight of the surficial soils, then the "critical gradient" may be exceeded. At that instant the soil can begin to blister, heave, and boil. This action, were it to occur during a flood event, would appear as flowing water from localized holes with both silt and fine sand particles being transported at these critical velocities. These can eventually appear as small sand volcanoes from the boiling source. If unabated, this piping action could concentrate from this disrupted region and progressively erode the underlying

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soils under the levee/floodwall system to the river's source. The piping may eventually trigger slope stability failure near the toe of the levee or nearby affected embankments due to a loss of soil support from piping occurring during prolonged periods of high underseepage resulting from a flood event. It is the responsibility of the Corps of Engineers to utilize design methods during the original preparation of the flood protection system that adequately evaluate this phenomenon. The original design and construction of the flood protection systems are intended to take this matter into account. The proposed railroad relocation is not intended to penetrate the underlying site soils; therefore, piping of soils should not be influenced by the railroad operation.

- Subgrade pumping locally from repetitive traffic loading: During periods of elevated ground water levels subgrade pumping of high-frequency traffic areas may occur. Soils which exhibit greater content of silt-sized particles are generally more prone to subgrade pumping issues. Soil subgrade that experiences repetitive traffic under high water level conditions, may be more prone to exhibit localized surface deformations, rutting from tires, or wave-like motion of the surface during vehicle or equipment operation. This pumping action is less evident when the repetitive loading is adequately distributed across the subgrade by means of an improved ballast and trackage system inherent to the railroad system. Limited deformation of the trackage is typically noticeable under normal railroad operation, generally returning to its near-original level once the trains have passed. Localized pumping of soils is not a feature of liquefaction activities, and therefore outside the realm of current evaluation. Displacements or possible subgrade pumping are relatively localized and should not impact adjacent structures along the alignment. Settlement of the trackage under normal repetitive operation (non-flood event conditions) is commonplace and is generally considered to improve the subgrade condition by densifying the soils and minimizing the potential for localized liquefaction concerns.
- Lateral flows, landslides, or slope stability concerns: Where the prevailing grade is generally more varied in profile than present at this project location, steep slopes or dramatic changes in grade may result. At such sites, vibration events could potentially create deep-seated effects upon the toe support of slopes, thus causing a concern for movement of the slope system. This current evaluation is not intended to address the potential of landslides initiated by the combined effects of elevated pore water pressures during flooding and train traffic at the Mankato, Minnesota site. The validity of this evaluation relies on the qualification that the ground surface along the railroad alignment is relatively level. The ground surface is assumed relatively level to a minimum distance of 15 feet on either side of the existing and proposed railroad track alignments.
- Soil liquefaction from vibration concerns under elevated water conditions: Liquefaction of soils generally requires a soil to be below the prevailing water table and during the same time period experience extreme repetitive vibration impacts, typically from a source like a major earthquake (M>5), blasting, or other dynamic event. The occurrence of this phenomenon, to date, in a railroad loading situation has never been recorded or documented to have occurred for a relatively level

trackbed condition, such as appears present at the current project site. In performing this review and evaluation of liquefaction potential for this site, it is also our assumption that railroad operations will temporarily cease in the event that the river level exceeds Elevation 779 feet (controlling low spot in the railroad track upstream of the downtown portion of the site). However, this evaluation conservatively assumes the water level at the ground surface near the floodwall and depot (Elevation 780 feet). If existing conditions or future plans for the site deviate from these assumptions, or other assumptions outlined throughout this document, consult with Burns & McDonnell for appropriate revisions to this letter report.

Soil Liquefaction

Liquefaction is generally considered an event where the soils temporarily lose their apparent in-place strength and briefly behave like a fluid. Liquefaction generally requires the soils to be saturated, below the prevailing ground water table, and exhibit a more cohesionless (predominately sand or silt-like particle size) consistency. The saturated, loose sandy soils then be loaded under a repetitive-cyclic vibration from a major force or event. The majority of all recorded instances of liquefaction have been identified to occur with earthquake events of magnitudes exceeding 5.0 by the Richter Scale. Generally, the smaller the event producing a seismic or vibratory loading, the shallower or shorter the impacted distance from the source of the event. To date, no apparent instances have been recorded where a railroad located upon relatively level grade has undergone liquefaction from rail operation.

The sudden drop of shear strength under undrained conditions from the yield strength to substantially smaller critical state is known as liquefaction. It may be triggered by the dynamic application of a single large increment of shear stress or by the repeated application of smaller shear stress increments and decrements by the ground shaking associated with an earthquake or explosion (seismic event). The loss of strength is so significant that the sand temporarily assumes the consistency of a heavy liquid. The damage is most severe when the liquefied zone is thick and the overlying confining mantle is relatively thin. The possibility of liquefaction is judged by comparing the anticipated dynamic shear stresses with the undrained shear strength of the sand at yield. [Terzaghi, Peck, and Mesri (1996)].

The factors that directly affect liquefaction potential of sandy soils include the soil grain-size distribution, in-place density, depth of prevailing ground water level, aspects to the seismic event (vibration magnitude and acceleration), location of drainage pathways for the release of water to the surface, stratigraphic layering of soils (such as confining clay layer over the sand), lateral and vertical dimensions of the potentially liquefiable deposits, magnitude and nature of superimposed loads and time period of sustained loading. Site factors which influence the overall density of the soil, and therefore have a correlative relationship to the potential for liquefaction are placement method of the soil formation (soil structure, as in alluvial-placed soils), previous soil-strain history, and potential for entrapped air. [Prakash]. Liquefaction typically occurs in loose, saturated, fine and uniform sands; with coarse sands less prone to this condition.

Liquefaction is also dependent on the nature, magnitude, and type of dynamic loading. Under steady-state vibrations, the maximum pore pressure develops only after a certain number of repetitive loading cycles have been imparted to the soil deposit (Seed and Lee, 1966). For instance in the extreme example of the deep-seated landslides which occurred in Anchorage during the Alaskan earthquake, these were triggered as a result of localized liquefaction about 90 seconds after the start of the major magnitude earthquake-induced ground motion (Seed and Idriss, 1971). Therefore, if the cyclic ground motion had lasted less than 90 seconds – say, 45 seconds – liquefaction would not have likely developed, and those respective slides would not have occurred. Also, the magnitude of the ground motion must be sufficient to develop significant shear stresses (and resulting pore pressures) capable of inducing liquefaction. Typically, liquefaction data has been developed from field observations of soils that have exhibited post-liquefaction effects, based primarily on earthquake records with Richter magnitudes ranging from approximately 5.0 to 8.5.

During a vibration event, the dynamic shear stresses induce pore pressures in the soil mass which increase with time (number of cycles) to a maximum pore pressure value. This maximum pore pressure value then remains constant before it starts to dissipate. The permeability and dimensions of the liquefiable deposit typically control the time-rate at which the dissipation of the pore water pressure may occur. Liquefaction may occur during this period of elevated pore water pressure, provided the vibration event produces sustained dynamic shear stresses of sufficient magnitude to increase the pore water pressure and subsequently decrease the effective confining overburden stress (and undrained shear strength) to zero. The confining overburden stress is directly dependent upon the prior noted factors of gradation, in-place density, depth of water level, and soil characteristics.

Seed and Idriss (1971) developed a simplified procedure for estimating the seismic shear stresses and the number of cycles needed to liquefy a soil for depths less than 50 feet in relatively level ground situations. The maximum shear stresses in the deposit are calculated as a function of the anticipated ground acceleration. Next the equivalent number of significant stress cycles (to reach the maximum pore water pressure) is determined. The number of significant stress cycles is dependent upon the duration of ground shaking and thus the magnitude of the earthquake (dynamic event). A determination of the stresses required to cause liquefaction is then made as a function of the deposit's relative density. Finally, the liquefaction potential is evaluated by determining whether the shear stress induced at any depth by the earthquake (dynamic event) is sufficiently large to cause liquefaction of the deposit at that depth. [Prakash].

Based upon our review of the literature regarding liquefaction of soils, it was noted that earthquake-induced liquefaction has rarely been evidenced for Richter Magnitudes below 5.0. A review of transit and railroad vibration literature also was performed; revealing no recorded documentation of past train-induced liquefaction encountered for level sites in the literature. Limited situations of suspected landslide failures either due to liquefaction of sand seams or due to softening of sensitive clays within railroad slopes were identified in the literature.

Characteristics of Train-Induced Vibrations

In general, vibration normally becomes a concern for one of the following reasons; either the amplitude of the vibration is large enough to cause excessive structural stress, or it is large enough to disturb the people in, or near the vibrating object. As far as most structures are concerned, vibration will disturb the people around the structure long before stresses become a problem.

A number of studies have been conducted previously for the subject site by others to estimate the impact of train-induced vibrations from the proposed expansion and upgrade of the DM&E Railroad. ESI Engineering, Inc. prepared a report dated October 28, 1999 that summarized work completed to assist the City of Mankato with an independent evaluation of the potential impacts and costs of vibration from the proposed DM&E project in Mankato. Wilson, Ihrig and Associates, Inc. prepared a report dated January 28, 2000 that provided a general evaluation of the potential impacts that may occur at a site (this general report did not specifically address the DM&E project site at Mankato). Wilson, Ihrig and Associates, Inc. also performed a more detailed evaluation of their site-specific vibration data collected in June 2001 from the Mankato site to assess the impact of the proposed train-induced vibrations on the surrounding structures. A draft version (dated September 10, 2001) of this report was submitted to Burns & McDonnell during final preparation of this letter report.

In addition to the above issues, a concern over the potential for localized liquefaction effects upon supporting soils to nearby foundation elements has been raised for this site. Typically, liquefaction studies evaluate the likelihood of soil support problems in the geotechnical area regarding major earthquake events. These are much more significant in the magnitude of energy released to a much more regional area. The more focused evaluation of liquefaction potential for localized train-induced vibrations is limited in the industry. Carter and Seed (1988) studied the liquefaction potential of train-induced ground vibrations for "loose" saturated sands. The authors identified these "loose" sands as typified by average corrected blow counts at or below 5 blows per foot [$(N_1)_{60}$ <= 5 BPF]. In their paper, they looked at the limited past recorded occurrences of liquefaction from blasting, pile driving, and train-induced vibrations. Further, they developed a methodology to evaluate the potential region that might be labeled "suspect" to liquefaction potential, based upon both a shear strain analysis and a shear stress analysis. Using the shear strain approach, the authors concluded that the ground vibrations generated by trains are not only incapable of liquefying level, loose sand deposits located at distances greater than 10 feet from the nearest rail, but they are also probably incapable of generating significant pore pressures within sands at these distances. Based upon an evaluation of liquefaction potential using the shear stress approach, the authors concluded that the ground vibrations generated by trains are probably incapable of liquefying level, saturated, loose sand deposits located at distances greater than 10 feet from the tracks in all but exceptional cases. It should be noted here that analyses were not performed at distances closer than 10 feet from the rail, due to apparent constraints of the Rayleigh Wave Theory. Such conclusions are consistent with the observation that no case histories are known to have been reported where ground vibrations generated by trains have liquefied level sandy sites. A review of the procedure indicates that the region representing "suspect" potential for liquefaction is very dependent upon the in-place density of the underlying soils, extent of soils below the water table, and in turn can be represented by a flattened elliptical-like arc that becomes progressively shallower to the surface the farther away from the track bed. Generally, for the "loose" soils represented in the authors' studies, the depth to which the "suspect" soil region lies below the surface is relatively limited.

The limited extent of technical literature addressing the occurrence of liquefaction upon level ground sites resulting from train-induced vibrations appears to indicate that the potential risk of liquefaction-related damage to adjacent structures and facilities is remote.

Site Susceptibility to Train-Induced Liquefaction
The basic methodology established for the general evaluation of liquefaction of soils relies upon earlier work presented by Seed and Idriss (1971) for earthquake-induced analysis. Two techniques, both stemming from the original Seed and Idriss (1971) work, were employed to evaluate the site susceptibility to soil liquefaction due to train-induced vibrations.

- Technique 1: The first technique employed for evaluation of the train-induced liquefaction was based directly on the general methodology developed by Seed and Idriss (1971). An attempt was made to correlate the magnitude of vibration caused by the train passage to an effective earthquake Magnitude, based on an equivalent mass energy or force induced by a dynamic event. The resulting magnitude was substantially lower than typical earthquake events causing soil liquefaction. The computer program *LIQUEFY2* was used to estimate the factor of safety against soil liquefaction versus depth. A more detailed discussion of the Technique 1 methodology and its results are provided below.
- Technique 2: The second technique employed for evaluation of the train-induced liquefaction was based on a modification of the basic Seed and Idriss (1971) liquefaction evaluation procedure. Pando, Olgun and Martin (2000) and Carter and Seed (1988) outline the required modifications to the basic liquefaction evaluation procedure for application to liquefaction potential from train-induced vibrations. The modified procedure develops a revised cyclic stress ratio curve based on an increased number of cycles potentially causing liquefaction from the train passage. The modified procedure also accounts for the effects of dry preloading on the behavior of the subsurface soils to resist liquefaction. A more detailed discussion of the Technique 2 methodology and its results are provided below.

Technique 1
To facilitate the analysis for Technique 1, a computer program designed for the evaluation of earthquake-induced soil liquefaction was the primary means of verifying the relatively low susceptibility for train-induced soil liquefaction at the Mankato site. The computer program *LIQUEFY2* was used to estimate the factor of safety against soil

liquefaction versus depth. A discussion of the parameters required in the evaluation is presented below. Prior geotechnical subsurface data from the US Army COE within the area of the floodwall was used to develop a design soil profile for the liquefaction evaluation at the site. This design profile is comparable to the soil profile information presented by ESI Engineering on Figure 4-1 of their DM&E Vibration Assessment Report dated October 28, 1999 (see Attachment 1). In general, the soil profile was conservatively assumed to consist of medium dense, saturated, fine sand with a D_{50} grain size value of 0.30 mm, 5% material passing the No. 200 sieve, and a minimum $(N_1)_{60}$ value of 10 blows per foot. These in-place unit weights are substantially denser than those used by other authors in the earlier reported literature by Carter and Seed (1988). In fact, the overall unit weights of the soils along the levee/floodwall system appear to exhibit higher overall densities, as noted by the range of blow counts from as low as 10 upwards to over 30 blows per foot.

Vibration monitoring performed at the site by Wilson Ihrig and Associates during June 2001 produced estimates of the peak ground acceleration due to the existing rail traffic near the floodwall of up to 0.015g. The vibration monitoring was performed at the base of the existing floodwall (approximately 37 feet from the existing rail line). Based on the generalized ground surface vibration attenuation curves from Figure 10-1 of DOT-T-95-16 (see Attachment 2), an approximate increase of 5 dB was estimated for a decrease in the train-to-floodwall distance from approximately 37 feet to 18 feet. The September 10, 2001 draft report from Wilson, Ihrig and Associates, Inc. indicates that the proposed track location and train speeds associated with the proposed DM&E Railroad upgrades in Mankato, Minnesota will result in a maximum horizontal ground acceleration of 0.135g at the floodwall location.

An earthquake of Magnitude 1.0 relates to a Seismic Energy Yield equivalent to 30 pounds of dynamite, which is comparable to a large blast at a construction site (Nevada Seismological Laboratory website). The Seismic Energy Yield corresponding to 1 pound of dynamite is typically greater than that associated with the current rail vibrations (0.05 in/sec at 37 feet from source) and proposed rail vibrations (0.5 in/sec at 18 feet from source) at the DM&E site (Figure 7 of Wiss, 1981 – see Attachment 3). Therefore, it can be reasoned that the train-induced vibrations at the Mankato site are equivalent to a Richter Magnitude of 1.0 or less, based upon energy release. For the liquefaction analysis, the input range of Magnitude value for the *LIQUEFY2* program has a lower limit of 5.0 (due to the scarcity of liquefaction data with Richter Magnitudes less than 5.0). An earthquake of Magnitude 5.0 relates to a Seismic Energy Yield equivalent to 32,000 tons of dynamite (Nevada Seismological Laboratory website). Therefore, the results of the *LIQUEFY2* model are based on a Seismic Energy Yield approximately 64 million times greater than the values estimated by correlation with Figure 7 of Wiss (1981) for the train-induced vibrations. As a result, the *LIQUEFY2* model may be considered by some to be somewhat conservative in its determination of the factor of safety against soil liquefaction from the anticipated train-induced vibrations.

The results of the Technique 1 analysis using the *LIQUEFY2* program indicate that soil liquefaction in the vicinity of the Mankato floodwall should only occur in cases where peak ground accelerations exceed approximately 0.30g (for a 5.0 Magnitude seismic event). Peak ground accelerations for the proposed railroad have been estimated as 0.135g at the floodwall, and the actual Magnitude has been estimated as less than 1.0. Therefore, it has been concluded from these results of the *LIQUEFY2* program that the potential for soil liquefaction and the resulting liquefaction-related impacts on wayside structures and land uses caused by the expansion and upgrade of the DM&E Railroad and introduction of unit coal trains in Mankato, Minnesota are considered to be low to non-existent. This is consistent with the limited technical literature relating to modeling procedures and evaluation of train-induced soil liquefaction for level ground sites.

Technique 2
The methodology outlined in Carter and Seed (1988) was also employed as Technique 2 for a secondary screening means of evaluating the liquefaction potential caused by train-induced ground vibrations at the Mankato, Minnesota site. The soil profile from the Mankato site [with $(N_1)_{60}$ =10] was used in conjunction with the Shear Stress Approach outlined in the Carter and Seed (1988) literature to estimate the approximate extent of potentially liquefiable soils within the vicinity of the proposed railroad improvements. As noted previously, the methodology to look at the "suspect" zone of potential liquefaction by Carter and Seed (1988) parallels the *LIQUEFY2* procedures, with the exception to the range of earthquake magnitudes and number of cyclic loading conditions resulting from the train-induced event. Based on interpretation of Figure 5.19 of Carter and Seed (1988), [with corrections to account for the improved blow count information at the Mankato site, the effects of prior dry preloading to the sand deposits based upon repeated loading under normal operation, and conservative assumption of water at ground surface of Elevation 780 feet] the "suspect" potentially-liquefiable zone was estimated to extend laterally to no greater than approximately 12 feet. It was also determined that the potentially-liquefiable zone extends to an approximate depth no greater than 2 feet at a corresponding distance of 10 feet away from the railroad track.

Summary
The projected "suspect" zone of marginally liquefiable soil is considered to be the extreme extent to which soil liquefaction may occur, and is not anticipated to impact wayside structures or land uses along the DM&E project alignment for several key reasons.

- The occurrence of soil liquefaction in close proximity to the existing railroad track has not been documented at the site; in fact, no prior case histories are known to have been reported where ground vibrations generated by trains have liquefied level sandy sites (Carter and Seed, 1988).
- Results of the Technique 1 evaluation indicate no wide ranging or extensive problems with liquefaction potential since the applied energy (and resulting equivalent Magnitude) of the train traffic is substantially lower than for typical liquefaction-causing earthquakes.
- Results of the Technique 2 evaluation approximate the extent of a representative "suspect" arc (surfacing at a region 12 feet from the tracks) where liquefaction

potential may exist. The method outlined in Pando, Olgun and Martin (2000) and Carter and Seed (1988) for this site should be considered as a screening procedure only, since the actual blow count values and in-place densities of the site soils generally appear to be better than assumed in the analysis. Areas beyond the "suspect" region should be considered to exhibit no potential risk of liquefaction. Final design of the facility may require a site-specific subsurface investigation along portions of the trackage to more accurately determine current unit weights and soil gradations along the proposed right-of-way. At which time, these refined soil parameters may be modeled at various regions along the river to further limit the lateral and vertical extent of potentially liquefiable soils.

- Even under a worst case scenario, the approximate extent of "suspect" liquefaction potential does not encroach upon or within near proximity to any known foundation soils under the floodwall or depot.
- The water table has conservatively been assumed at the ground surface, although operation of the train will likely be discontinued prior to the river attaining this level near the floodwall reach. To that issue, train operation will cease once the upstream river elevation reaches Elevation 779 feet (at a low point in the rail line assumed to be approximately 5 miles up-gradient from the depot location), per prior experience related to our offices. Assuming the river's surface gradient slopes downstream toward the floodwall/depot/gauging station at approximately 1.0-foot/mile, the river elevation exterior to the wall would approximate Elevation 774 feet when all train activity ceases. During final design, if the need is subsequently determined to limit or restrict either train speed or overall operation, then a criteria can be established correlating these restrictions to a flood-related river elevation which triggers the potential for liquefaction at "suspect" locations along the project alignment. In effect, if the soils that are identified as liquefaction prone under train-induced vibrations are not allowed to be loaded when the water table intrudes upon this pre-defined elevation, then concern for localized liquefaction both under the track bed and to any limited "suspect' region beyond is effectively eliminated.

This evaluation has shown that the foundation elements of the floodwall, depot, and facilities beyond 12 feet from the rail line are well beyond any influence region to where liquefaction potential is remotely suspect under the conditions reviewed. This limited influence region is likely conservative in its current delineation, since the evaluation was based upon the minimum observed blow counts from limited available borings provided by the Corps of Engineers from other project activities as well as other conservative parameters. A site-specific subsurface investigation at the time of final design would allow for refinement of this potential influence region, with the likelihood of substantial reduction of its lateral and vertical extent. An important point in the evaluation of liquefaction is the need for soils to be below the prevailing water table when undergoing repeated cyclic loading. Train operations will be discontinued once the low water point on the line upstream from the floodwall region achieves Elevation 779 feet. Based upon typical flow characteristics of the river, both the elevations of the river and the underlying ground water downstream at the floodwall reach should be less than 774 feet. Therefore, the soils that are suspect to liquefy should not be submersed below the water table at the time that railroad usage is curtailed within the floodwall region. As a result, if the soils are not saturated within the "suspect" region, any concerns for adverse shallow deformations resulting from the limited influence zone should be virtually eliminated. Discontinued operation of the rail associated with this upstream event appears to provide additional safeguards that may minimize the need for track re-leveling and may provide other beneficial secondary impacts to the surficial soils at the site.

In closing, it should be noted here that this letter report is intended to address only the soil liquefaction aspects of the train-induced ground vibration study. The remainder of the ground vibration impacts associated with the proposed unit coal trains on the DM&E Railroad are being addressed by others.

Attachments:

1: Figure 4-1 of the ESI Engineering Vibration Assessment Report (1999)
2: Figure 10-1 of DOT-T-95-16
3: Figure 7 of Wiss (1981)
4: References

References

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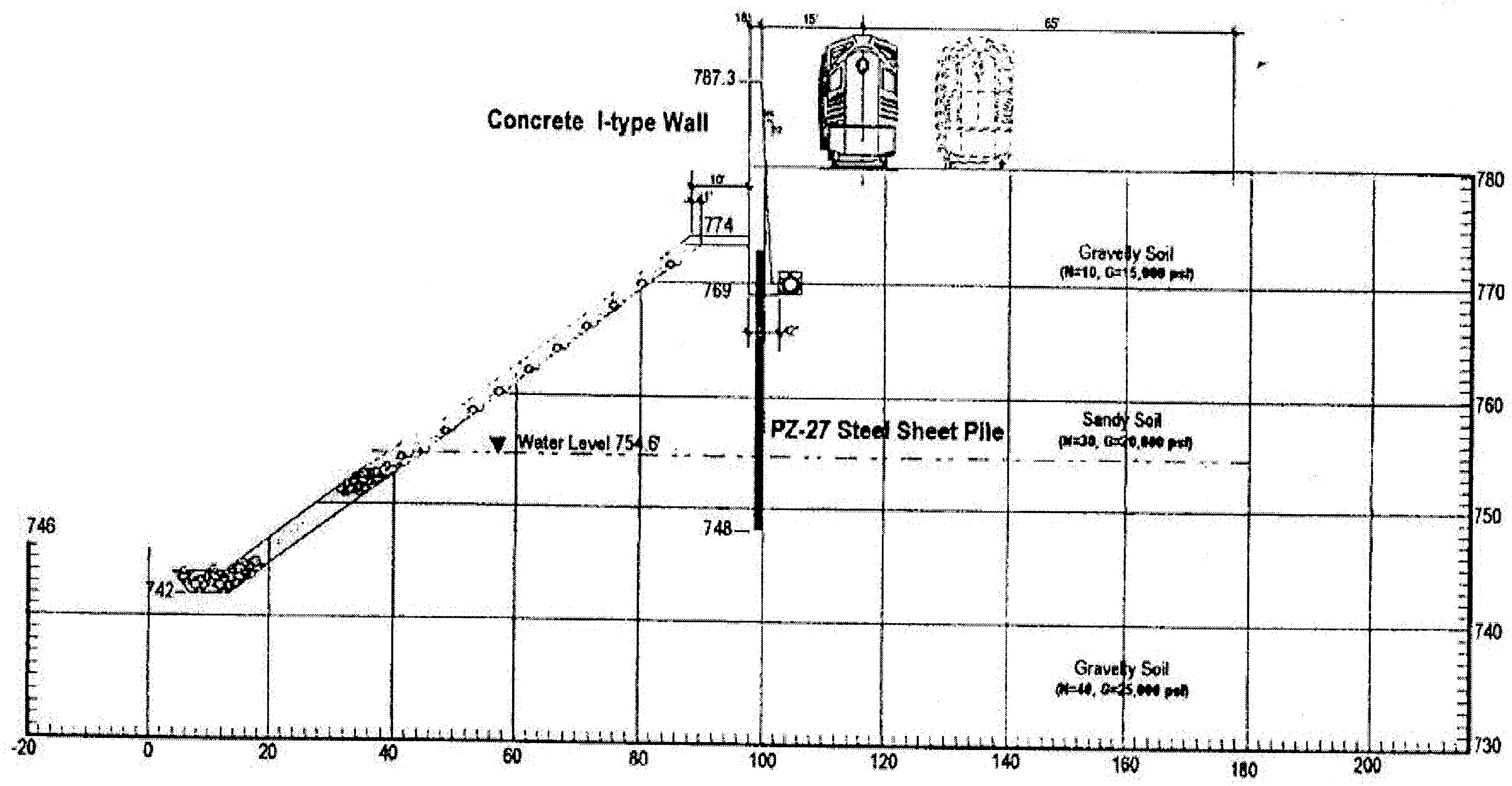


Figure 4-1 – Floodwall at Section H-H at Depot (Station 86+50)

Burns & McDonnell
SINCE 1898

Attachment 1
Adapted from ESI Report
(Dated October 28, 2001)

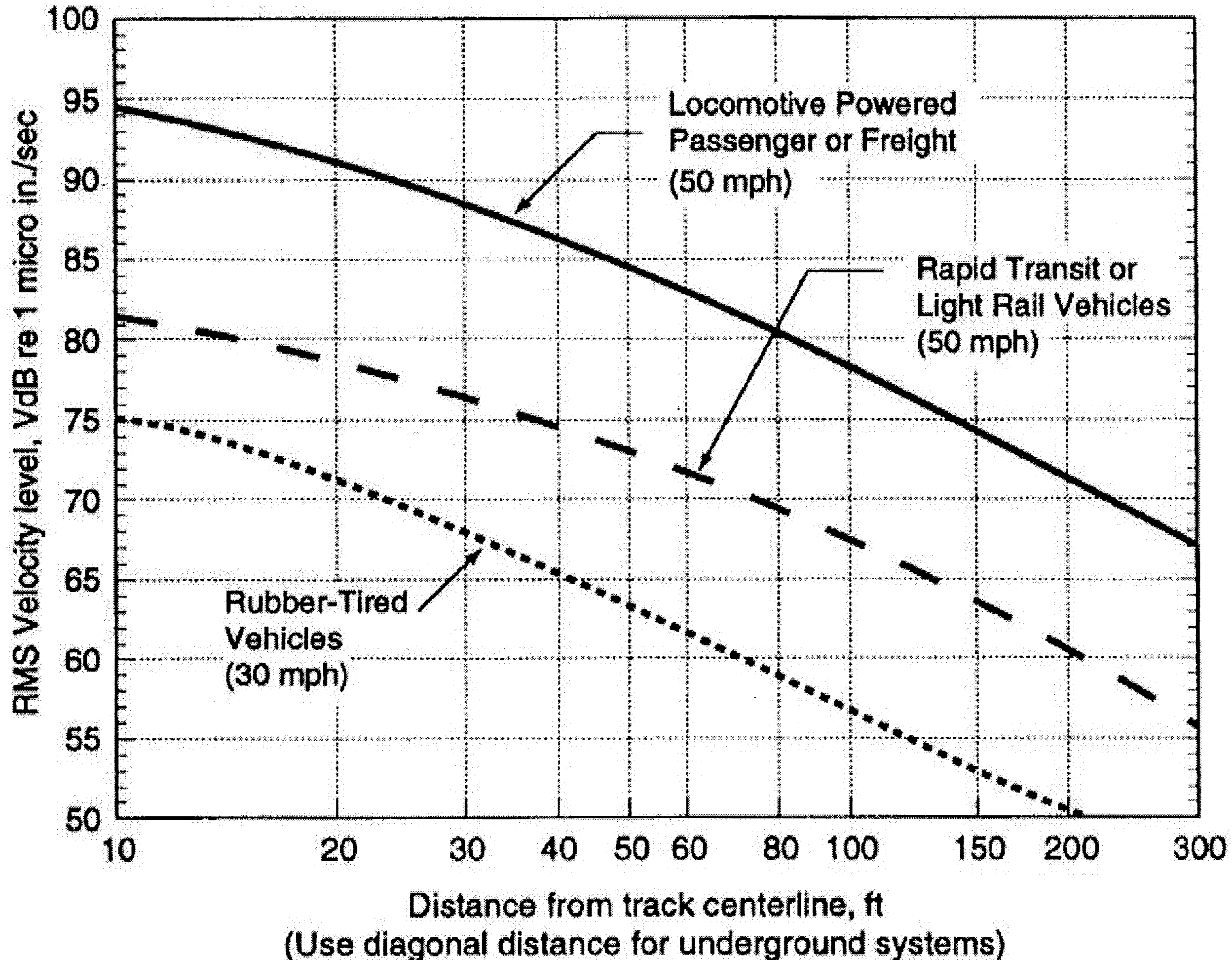
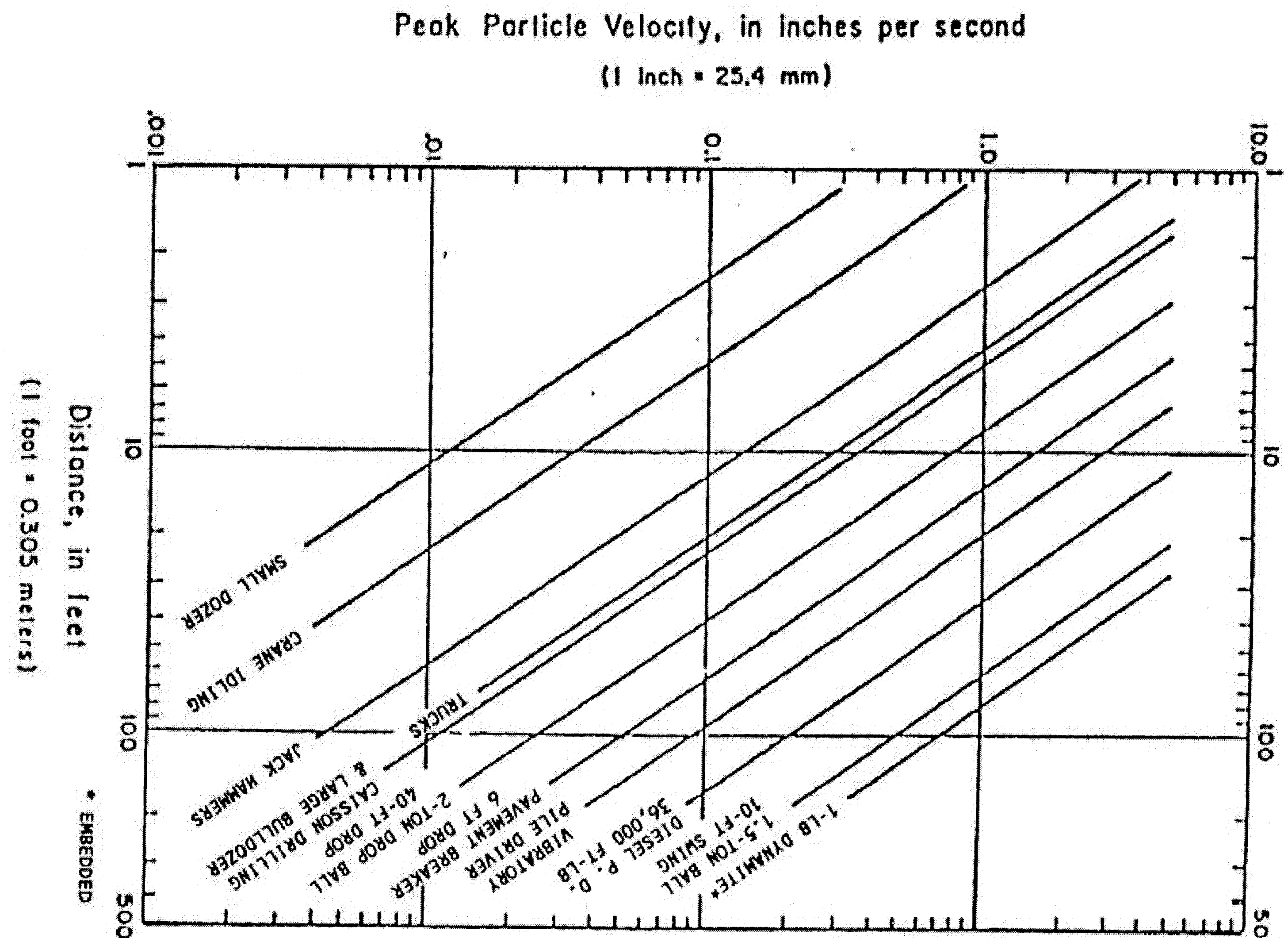


Figure 10-1 Generalized Ground Surface Vibration Curves

Burns & McDonnell
SINCE 1898

Attachment 2
Adapted from DOT-T-95-16



Burns & McDonnell
SINCE 1898

Attachment 3
Adapted from Fig. 7, Wiss (1981).